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RESEARCH

Analytical Performance Assessment of Deteriorated Prestressed Concrete Beams

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Abstract

In this paper, a new approach is developed to numerically evaluate the structural performance of deteriorated prestressed concrete beams using the finite element analysis application RCAHEST (Reinforced Concrete Analysis in Higher Evaluation System Technology). A prestressing steel element was modified to represent the interaction between deteriorated concrete and prestressing steel. Considering the corrosion effects—as shown by the reduction of the prestressing steel section and loss of bond—a nonlinear material model was proposed for deteriorated prestressed concrete behavior. The modified damage index is intended to provide a numerical method for quantifying the structural performance of deteriorated prestressed concrete beams. The analytical procedure developed for the performance evaluation of deteriorated beams was validated by comparison with the credible test results, thereby enabling a more reliable and rational prestressed concrete beams design process.

Keywords Performance, Deteriorated, Prestressed concrete, Beam, Modified damage indices

1 Introduction

Prestressed concrete (PSC) structures often reside in corrosive environments—including oceans and salinized or deicing salt environments—where the prestressing system corrodes easily. Numerous resources have been devoted to the repair and rehabilitation of deteriorating PSC members.

Carbonation of concrete, alkali-silica chemistry, and corrosion of steel reduce the strength and stiffness of PSC structures due to long-term environmental effects. As corrosion processes, not only does the mass of the steel decrease, but the ductility of the material decreases, leading to brittle failure (Du et al., 2005; Franceschini et al., 2022; Hanjari et al., 2011; Jeon et al., 2019; Shaikh, 2018; Tapan, 2007; Xu et al., 2021, 2022; Yanaka et al., 2016).

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In addition, many efforts have been made in understanding the deterioration of PSC structures. However, only limited studies (Coronelli et al., 2009; Dai et al., 2016; Ramseyer & Kang, 2012) have been done on the performance assessment of deteriorated PSC beams.

In PSC structures, the potential consequences of steel corrosion are severe than in reinforced concrete (RC) structures because prestressing steel is subjected to high stresses, when combined with cross-section reductions and notch effects can be fatal to safety. Pitting corrosion typically occurs in the case of natural corrosion, which results in the localization of strain and stress, leading to strand breakage. In particular, the bond loss between prestressing steel and concrete can decrease the prestressing force, the deterioration of the steel leading to the brittle failures (ACI 222.2R-01, 2001; Li et al., 2017; MacDougall & Bartlett, 2002; Rinaldi et al., 2010; Wang et al., 2014).

This study aims to provide insight into the numerical performance assessment, as experimental assessment of their structural performance can be expensive and time consuming.

Consequently, the author developed a numerical evaluation method for deteriorated PSC beams using modified damage indices. RCAHEST, a nonlinear analysis application used for assessing concrete structures, was used (Kim, 2019, 2022; Kim et al., 2003, 2005, 2007).

To evaluate the ability of RCAHEST to forecast the structural performance of deteriorated PSC beams, computational results using the modified application were compared to reliable experimental results by Rinaldi et al. (2010).

2 Nonlinear Finite Element Analysis Application, RCAHEST

The modified computer application, RCAHEST (Kim, 2019, 2022; Kim et al., 2003, 2005, 2007), was developed around an application shell called FEAP (Taylor, 2000). FEAP features a modular structure, as well as facilities for introducing custom finite elements, input utilities, and procedures.

The authors tried to implement such RC plane stress element and prestressing steel element which is newly developed to represent the interaction between concrete and prestressing steel.

The finite element model for PSC consists models that describe the inelastic behaviors of concrete, reinforcing and prestressing steels (see Fig. 1).

Concrete models can be classified as cracked concrete models and isotropic uncracked concrete models. The elasto-plastic and fracture model (Maekawa & Okamura, 1983), which is widely used for biaxial stress state, is used for uncracked concrete. For cracked concrete, three models were used to represent the concrete behavior (see Fig. 2), the basic model used to represent cracking is the

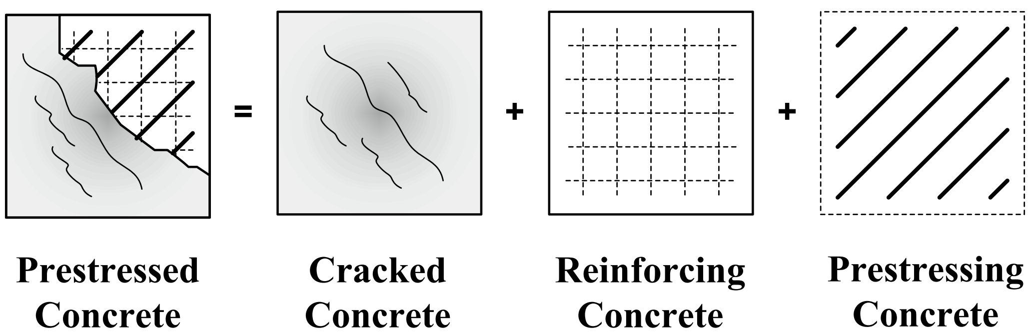


Fig. 1 Nonlinear material model for prestressed concrete

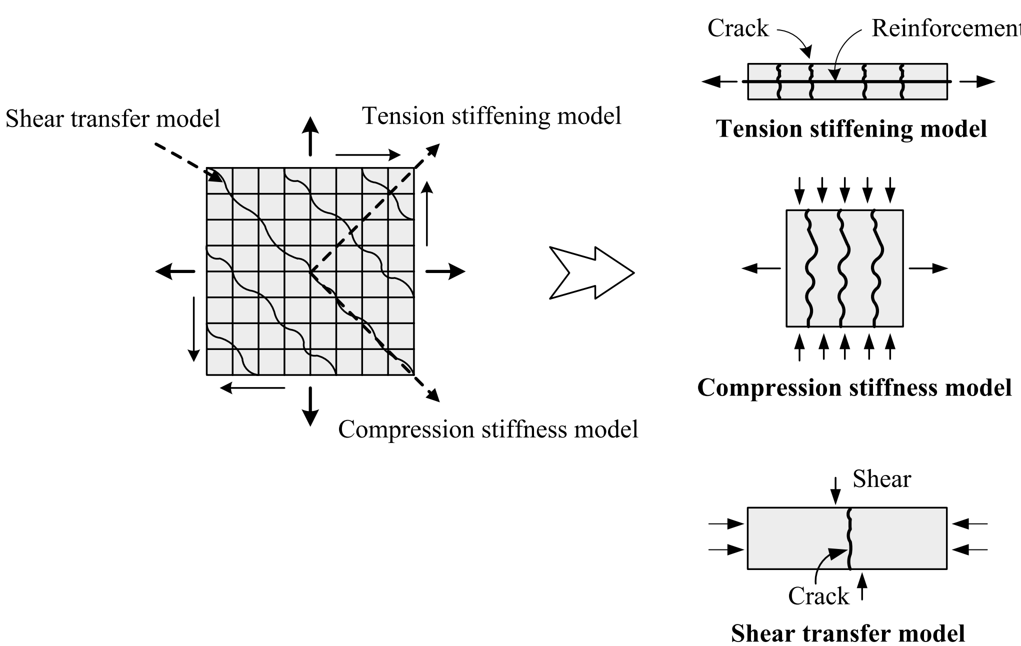


Fig. 2 Construction of cracked concrete model

non-orthogonal fixed-crack model using the concept of smeared crack.

By converting the tensile stresses of the concrete into a component perpendicular to the crack, an accurate tensile stiffening model can be obtained, especially when there is a significant difference in the ratio of reinforcing bar in the orthogonal direction or when the reinforcement is distributed in only one direction. A modified elasto-plastic fracture model was used to represent the compression behavior of the concrete struts between cracks in the crack plane direction. The model represents the degradation of the compressive stiffness by modifying the fracture parameters with strain perpendicular to the crack plane. To account for the effect of shear stress transfer due to aggregate interlocking on the crack surface, a shear transfer model based on the density function of the contact surface was used, assuming that the contact surfaces react elasto-plastically.

The stresses acting on rebar embedded in concrete are not uniform, with stress values being maximized at points where the rebar is exposed to the crack plane. If the stress–strain relation is in the elastic range, the constitutive law of bare reinforcement can be used. The post-yield constitutive method for reinforcement in concrete considers the bond properties, and the model is bilinear.

3 Modified Nonlinear Finite Element Model

3.1 Model for Prestressing Steel

The stresses acting on the prestressing steel embedded in the concrete are not uniform, and the greatest stress values at locations where the steels are exposed to the crack plane. If the stress–strain relation is in the elastic range, the constitutive equation for bare prestressing steel can be used.

The post-yield relation of the reinforcement in concrete is based on consideration of the bond properties (Kim et al., 2003). Bilinear diagrams used to describe the behavior of mild steel shows rough yielding, but cannot be immediately extrapolated to prestressing steel, and a multilinear approximation method may be required for prestressing steels without an explicit yield point. Therefore, the formulation was modified (see Fig. 3), the stress–strain relation for prestressing steel with bond effect was used, as follows:

$$f_{\mathrm{pt}} = f_{\mathrm{py}} + E_{\mathrm{ph1}}\left(0.03 - \varepsilon_{\mathrm{py}}\right) + E_{\mathrm{ph2}}\left(\varepsilon_{\mathrm{pu}} - 0.03\right), \tag{1}$$

$$E_{\mathrm{ph1}} = \frac{f_{0.03} - f_{\mathrm{py}}}{0.03 - \varepsilon_{\mathrm{py}}}, \tag{2}$$

$$E_{\mathrm{ph2}} = \frac{f_{\mathrm{pu}} - \mathrm{f}_{0.03}}{\varepsilon_{\mathrm{pu}} - 0.03}, \tag{3}$$

where f_{pt} = the prestressing steel stress; f_{py} = the prestressing steel yielding strength; f_{pu} = the prestressing steel ultimate strength; $\varepsilon_{\mathrm{py}}$ = the prestressing steel yielding strain; $\varepsilon_{\mathrm{pu}}$ = the prestressing steel ultimate strain; and $E_{\mathrm{ph1}}, E_{\mathrm{ph2}}$= the strain hardening rates.

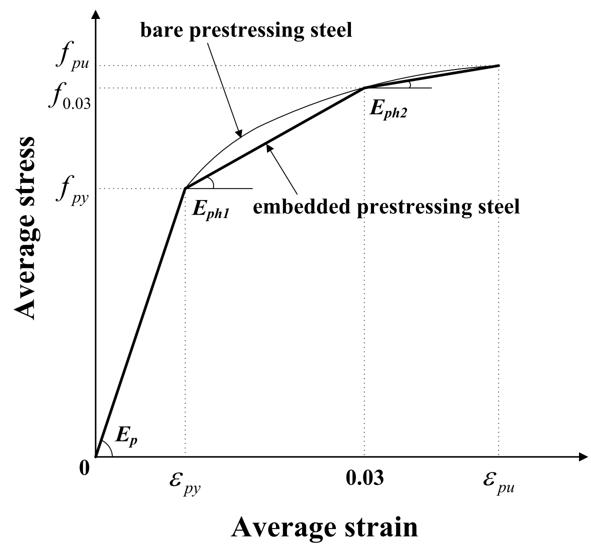


Fig. 3 Model for prestressing steel in concrete

Moreover, the author has provided a complete description of finite element model for concrete structures (Kim, 2019, 2022; Kim et al., 2003, 2005, 2007).

3.2 Deterioration Modeling in Prestressed Concrete

The deteriorated model proposed considered both localized and uniform corrosion and included a reduction in the bond strength and cross-sectional area (Bastidas-Arteaga, 2018; Dai et al., 2016; Du et al., 2005; Fernandez & Berrocal, 2019; Franceschini et al., 2022; Jeon et al., 2019; Song et al., 2019).

The two main forms of corrosion in prestressing steels are stress corrosion cracking and pitting corrosion, the pits reducing the steel cross-sectional area and increasing stress, which can lead to brittle fracture (Coronelli et al., 2009; Lignola et al., 2012; US Federal Highway Administration, 2000).

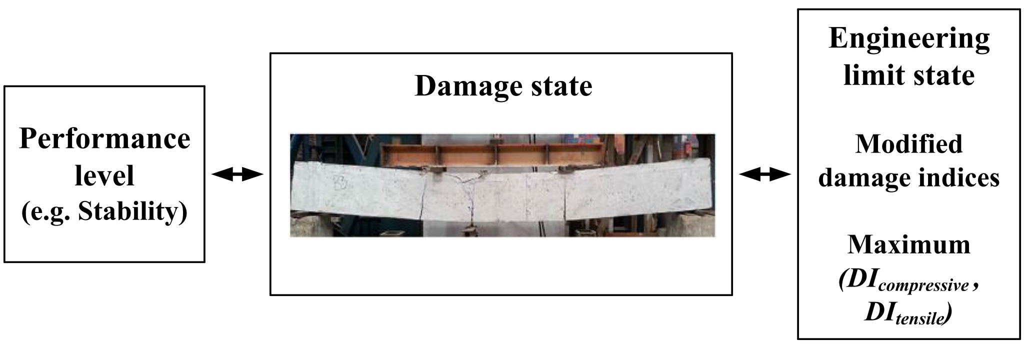
A model developed by Bhargava et al. (2007) was used in the nonlinear finite element model to evaluate corroded steel and concrete deterioration (Kim, 2022), the equations derived by conducting experimental tests on corroded RC specimens being modified as follows:

$$R_p = 1.0 \qquad \text{for } C_p \le 1.5\%, \tag{4}$$

$$R_p = 1.192\mathrm{e}^{-0.117C_p} \qquad \text{for } C_p > 1.5\%, \tag{5}$$

$$C_p = \frac{\Delta W_p}{W_p} \times 100, \tag{6}$$

Performance level	Service	Repair	Damage State	Damage Index
Fully operational	**Fully service**	**Limited epoxy injection**	**Hairline cracks**	$\leq$ **0.1**
Delayed operational	**Limited service**	**Epoxy injection, concrete patching**	**Open cracks, concrete spalling**	$\leq$ **0.4**
Stability	**Not useable**	**Replacement of damaged section**	**Bar buckling/fracture, core crushing**	$\leq$ **0.75**



Material	Type of failure	Failure criterion (ε_{cu} or ε_{tu})	Modified damage index ($DI_{compressive}$ or $DI_{tensile}$)
Concrete	**Compressive and shear**	$0.004+\dfrac{1.4\rho_s f_{yh}\varepsilon_{sm}}{f'_{cc}}$	$1-\Phi_c\left(\dfrac{2\varepsilon_{cu}-\varepsilon_{cs}}{2\varepsilon_{cu}}\right)^2$
Steel	**Tensile**	**0.10**	$1.20\left(\dfrac{\varepsilon_{ts}}{2\Phi_r\varepsilon_{tu}}\right)^{0.67}$

$DI_{compressive}$ = compressive damage index; $DI_{tensile}$ = tensile damage index

f_{yh} = yield stress of the confining steel; f'_{cc} = confined concrete compressive strength

ε_{cs} = compressive strain in analysis step; ε_{cu} = ultimate strain of concrete

ε_{sm} = steel strain at maximum tensile stress; ε_{ts} = tensile strain in analysis step

ε_{tu} = ultimate strain of reinforcing steel; ρ_s = transverse confining steel ratio

Φ_c = fatigue parameter for concrete; Φ_r = fatigue parameter for reinforcing steel

Fig. 4 Analytical performance assessment using modified damage indices

where R_p = the ratio of the bond strength of corroded prestressing steel to that of non-corroded prestressing steel; C_p = the corrosion level; ΔW_p = the average mass loss of corroded prestressing steel; and W_p = the mass of non-corroded prestressing steel.

The area A_{pc} of the corroded prestressing steel can be calculated by next equation:

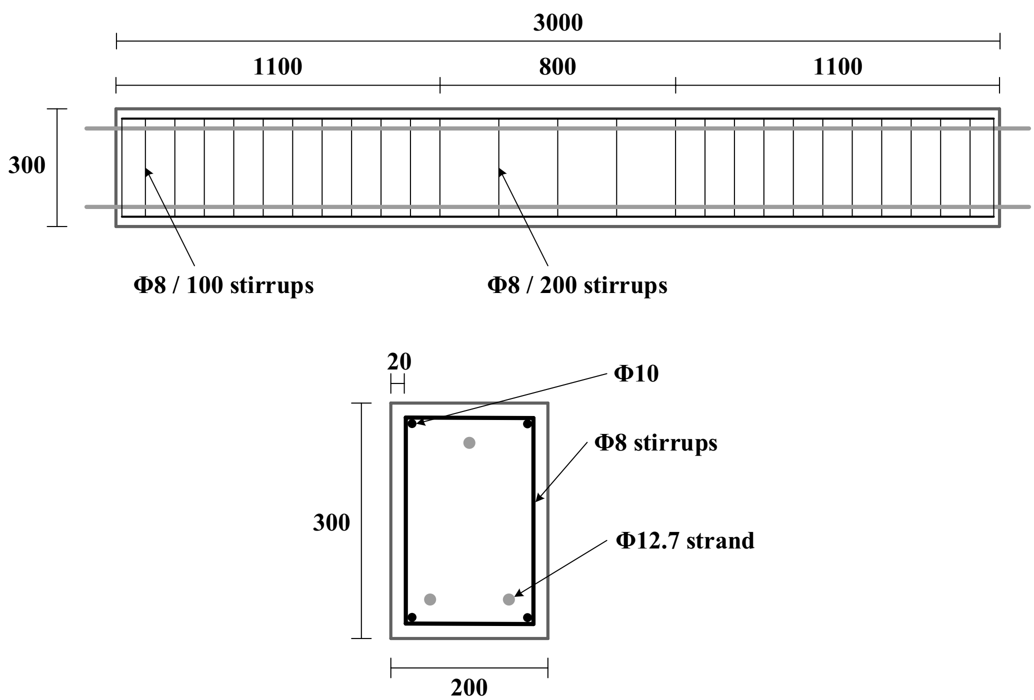


Fig. 5 Beam geometry (dimensions in mm) (Rinaldi et al., 2010)

Table 1 Properties of deteriorated test specimens (Rinaldi et al., 2010)

Series	2			3		
Average concrete cubic strength (MPa)	41.5			47.4		
Yielding stress (MPa)	1788					
Ultimate stress (MPa)	1976					
Nominal ultimate strain (%)	1.75					
Initial prestressing strength (MPa)	1300					
Beam No.	2	3	1	4	6	5
Corrosion level (%)	0	14	20	0	7	20

$$A_{pc} = A_p\left(1 - 0.01C_p\right), \tag{7}$$

where $A_p =$ the non-corroded prestressing steel area.

4 Analytical Performance Assessment of Deteriorated Prestressed Concrete Beams Using Modified Damage Indices

For the first time, an analytical performance assessment process using damage indices was proposed to evaluate the seismic performance levels of RC bridge piers. Explicit descriptions of the various performance levels are defined using the proposed criteria (Kim et al., 2007).

The widely used damage index such as Park and Ang (1985) is focused on the relation between the structural states and damage indices. Estimating damage indices at the structural level is a method to quantitatively assess structural damage, but it has an additional calculation process that is inconvenient. The element-level damage indices are calculated from the strain at the point of Gaussian integration calculated by the nonlinear analysis of each element, thus reducing inconvenience.

Damage indices were modified from a numerical parametric study of deteriorated PSC beams using nonlinear analysis. The analytical parametric study was done to examine the effects of the concrete strength, reinforcing and prestressing steel strength, size and tie spacing.

Figure 4 provides descriptions of the three structural performance levels that may be associated with deteriorated PSC beams. At the fully operational level, almost no damage exists and no repair is required. At the delayed

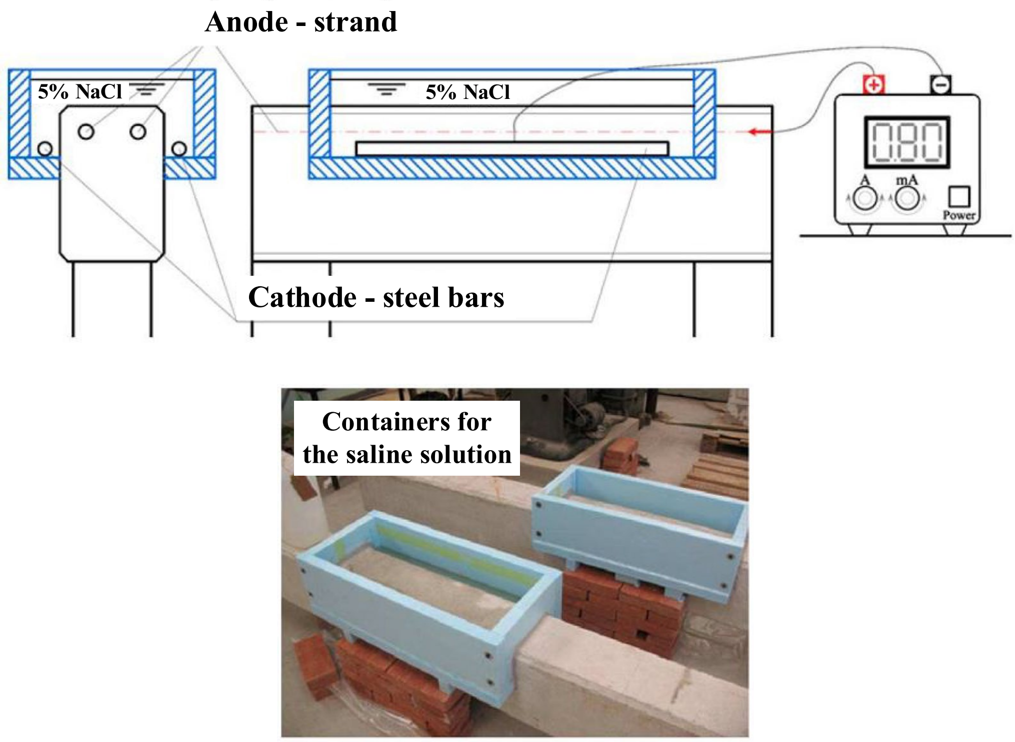


Fig. 6 Corrosion process (Rinaldi et al., 2010)

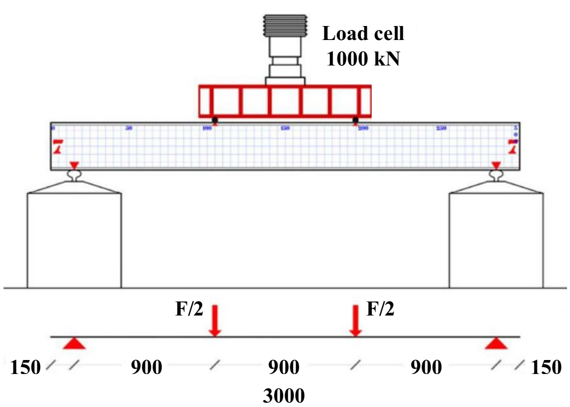


Fig. 7 Test set-up (dimensions in mm) (Rinaldi et al., 2010)

operational level, full utilization could be compromised, which could require repair. Finally, at the stability level, considerable damage requires a full or partial replacement. Kim et al., (2007) and Kim, (2019) provided a full description of analytical performance evaluation using the damage indices.

5 Numerical Examples

The experimental data for the deteriorated PSC beams obtained by Rinaldi et al., (2010) were used to verify the developed assessment method.

5.1 Description of Deteriorated Prestressed Concrete Beam Specimens

Six prestressed beams were artificially corroded and subjected to four-point bending test. Figure 5 shows the beam specimen details and steel arrangement. Prestressing is provided by 12.7 mm seven-wire strands, one located at the top of the section and two located at the bottom. Epoxy-coated reinforcing steels are used for corrosion, with Table 1 listing the properties of the prestressing steels and concrete.

Two bottom prestressing steels were subjected to artificial corrosion as these prestressing steels were in the tension zone. Special handmade polystyrene containers were filled with a 5% NaCl saline solution (see Fig. 6).

In each series, one beam was used as a reference and maintained under normal environmental conditions (Beam Nos. 2 and 4). Three levels of corrosion were employed: low level, mild level, and severe level (7%, 14% and 20% steel mass loss, respectively). All specimens

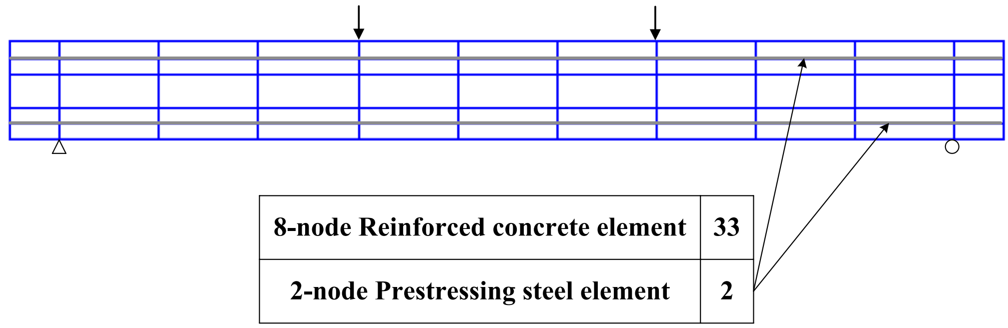


Fig. 8 Finite element model for deteriorated prestressed concrete beams

were subjected to a 1000-kN electromechanical jack, as shown in Fig. 7. Specifically, strands located in areas of the beam where corrosion was advanced (approximately 700 mm) were cut, removed from the specimen, and cleaned. Finally, it was weighed and compared to the non-corroded specimen to find the mass loss (Rinaldi et al., 2010).

5.2 Description of the Finite Element Model and Analytical Results

Figure 8 represents the boundary conditions and finite element discretization of the specimen. The finite element model comprises an eight-node RC plane stress element and a two-node prestressing steel element. By assuming a uniform corrosion model for the prestressing steel and a penetration depth, the cross-sectional area of the corroded steel can be calculated.

Figures 9a and 10a show the load–displacement curve at the mid-span of the undamaged beam. The developed nonlinear model successfully predicts the load–displacement relation of a non-damaged beam. Figures 9 and 10 also show the analytical and experimental load–displacement relationships of damaged beams exhibiting flexural failure with rupture of the prestressing steels.

The predicted results for the beam specimens showed that the ratios of experimental to numerical ultimate strength averaged 0.98 and the CV was 4% (see Table 2).

5.3 Design Parameter Studies

The load–displacement curves for Series 2 are presented in Fig. 11. The peak load sharply decreases from Beam No. 2 to Beam No. 3 and No. 1. In Beams No. 3 and No. 1, the gradual breaking of the wires is characteristic of post-peak behavior. Both the numerical and experimental results indicate that mild and severe levels of corrosion result in shear strength losses of approximately 55% and 65%, respectively.

The load–displacement relation for Series 3 is shown in Fig. 12. Baseline Beam No. 4 fails under a load of 210.0 kN. This value is higher than that of Beam No. 2, owing to its higher concrete strength. Beam No. 6 fails under a load of 200.0 kN with minor corrosion. For modest corrosion loss, Beam No. 5 has an ultimate strength of 70.0 kN (a 66.0% reduction). The analytical results are 207.9, 198.1 and 70.6 kN, respectively. Severe corrosion loads greatly reduce the ultimate strength of the corroded PSC beam. Owing to the limited level of corrosion, Beam No. 6 exhibits a unique behavior, the test results indicating that brittle failure occurs because of concrete and wire breakages. The ductility is questionable as the load–displacement curve flattens, and the pre-tensioning reinforcement partly breaks and plasticizes.

The sample of crack mode at failure is characterized by near-vertical cracks with branches (see Fig. 13a, b). Figure 13c also shows the trend of the modified damage index, which indicates a gradual progression of damage.

Failure modes for PSC beams are different. For example, the failure modes of Beam No. 6 are similar to those of Beams No. 2 and No. 4, their failure being due to the concrete crushing in the extreme compression fibers rather than via wire breakages. While the corrosion levels are severe, the failure of Beam Nos. 3, 1 and 5 is initiated by wire rupture at the position of maximum corrosion loss, accompanied by a decrease in the load-carrying capacity. In other words, the failure mode of PSC beams changes from concrete crushing to corrosion wire rupture as the corrosion level increases.

The ultimate behavior of beams with severe corrosion due to wire rupture was found to be equivalent to conventional reinforcement with only one wire. This result is consistent with experimental results showing that among strands removed from beams with 20% corrosion, only the inner strands were not damaged and very often the outer strands were damaged.

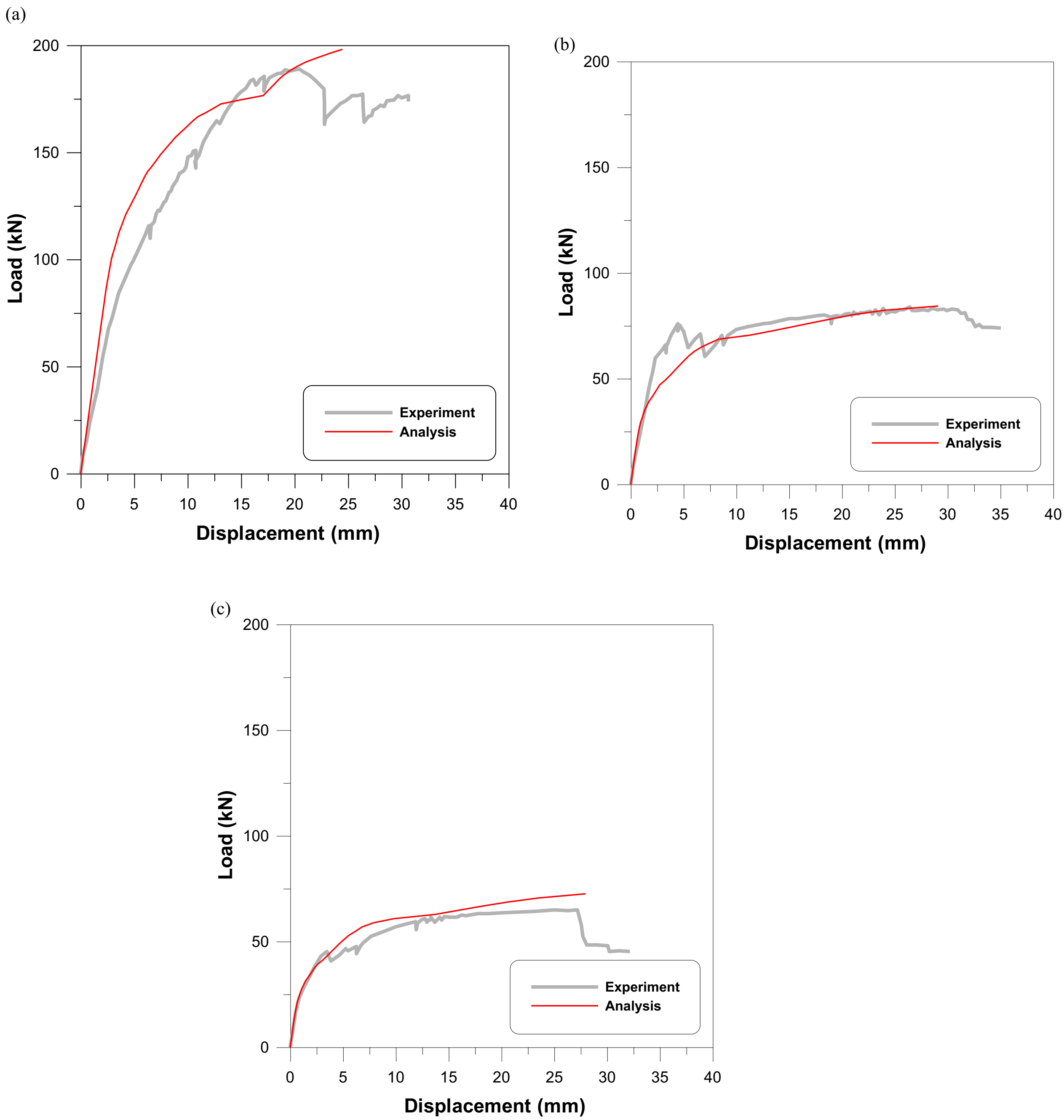


Fig. 9 Load–displacement curves for Series 2: (**a**) Beam No. 2, (**b**) Beam No. 3 and (**c**) Beam No. 1

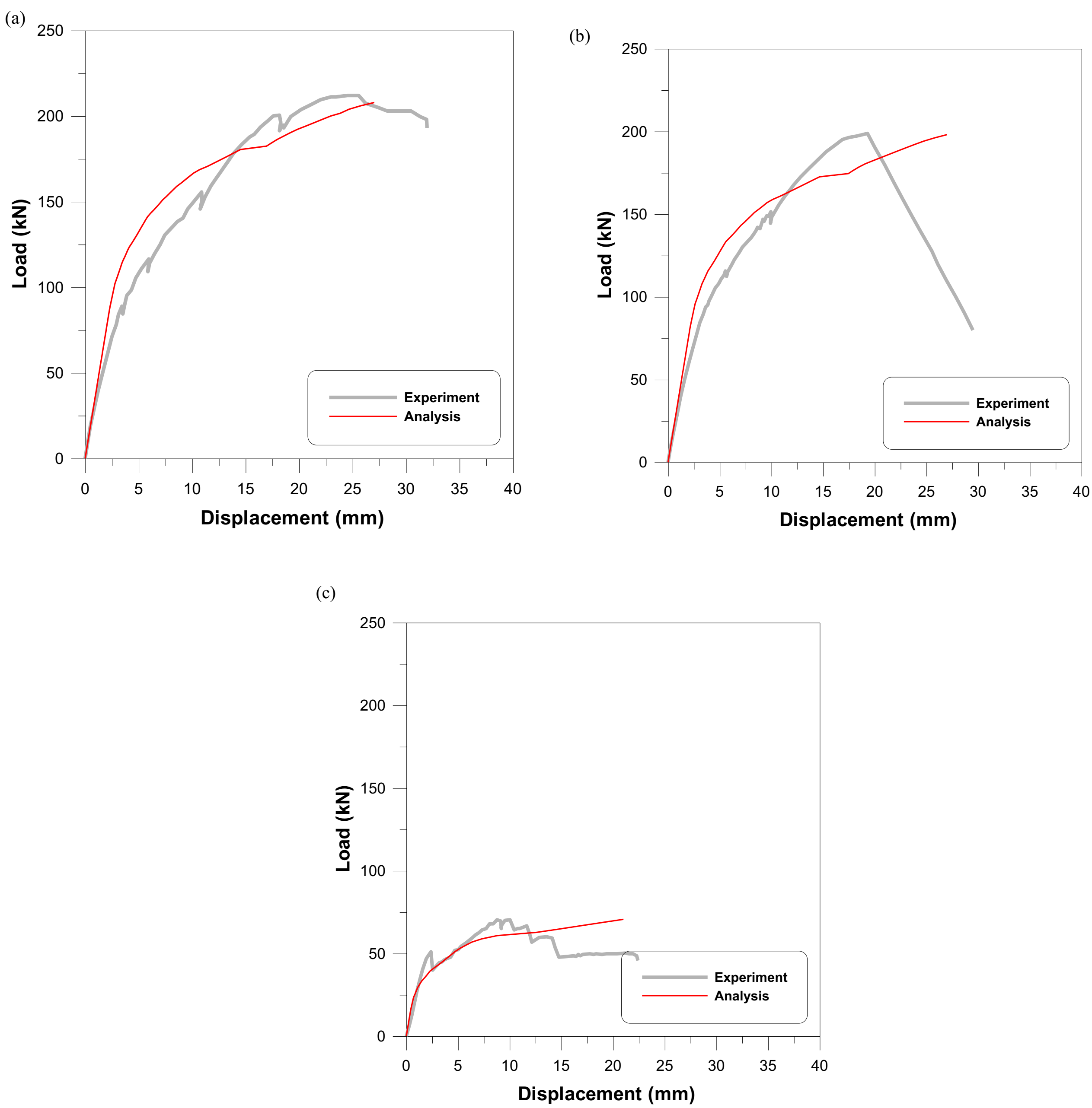


Fig. 10 Load–displacement curves for Series 3: (**a**) Beam No. 4, (**b**) Beam No. 6 and (**c**) Beam No. 5

Table 2 Experiment and analysis results

Beam No.	Experiment V_{max} (kN)	Analysis V_{max} (kN)	Ratio of experimental and analytical results
2	190.0	198.1	0.96
3	85.0	84.3	1.01
1	66.0	72.6	0.91
4	210.0	207.9	1.01
6	200.0	198.1	1.01
5	70.0	70.6	0.99
Mean			0.98
COV			0.04

V_{max}= maximum shear force

The agreement between the analytical and experimental results verifies the developed method and its ability to encapsulate the corrosion degradation mechanisms. Moreover, the structural performance level of deteriorated PSC beams can be accurately predicted using the proposed analytical assessment procedure, thereby enabling a more reliable and rational PSC beams design process.

6 Conclusions

A numerical study was performed to quantify structural performance of deteriorated PSC beams with modified damage indices. The following conclusions were drawn from the results of the numerical study:

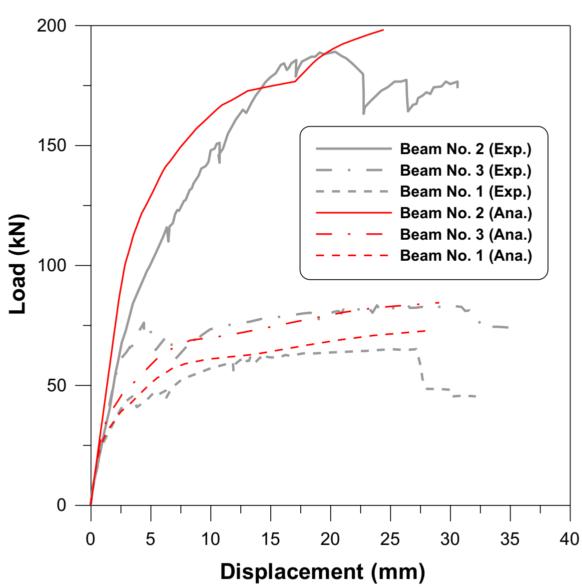


Fig. 11 Comparison between experimental and analytical results for Series 2

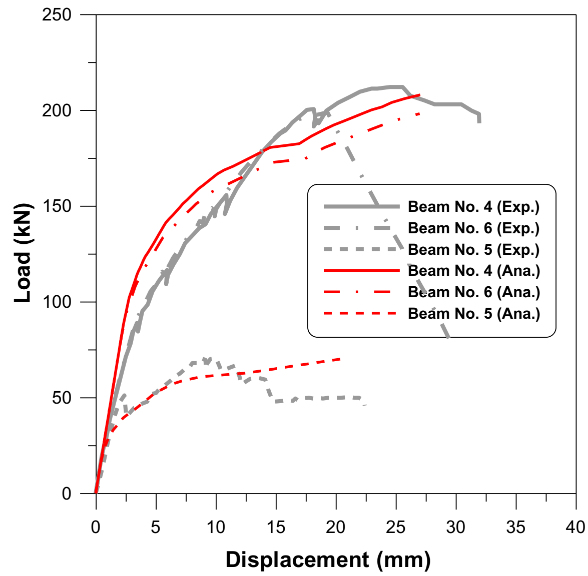


Fig. 12 Comparison between experimental and analytical results for Series 3

(1) A prestressing steel element and 2D RC plane stress element for the nonlinear analysis of concrete structures exposed to corrosion were presented. The proposed formulation permits modeling the damage effects of uniform and pitting corrosion in terms of a reduction in the cross-sectional area of corroded steel, reduction of ductility of steel, deterioration of concrete strength and spalling of concrete cover.
(2) The developed analytical method and detailed investigation results of deteriorated PSC beams can improve our understanding of the deterioration effects. The finite element model also gives a tool that could be used to develop a better understanding of the damage propagation mechanisms caused by deterioration.
(3) The corrosion of prestressing steels greatly affects the behavior of simply supported PSC beams in terms of both failure modes and load-bearing capacity. The effects of prestressing steel corrosion on the load–displacement behavior and failure modes also depend on the corrosion levels.
(4) The modified damage indices provide a quantitative prediction of the shear strength of deteriorated PSC beams, which could assist in establishing of cost-effective repair and rehabilitation system. The proposed analytical procedure could be used to assess the actual structural performance of the deteriorated PSC structures.
(5) Additional parametric studies are needed to confirm the design under practical conditions for use

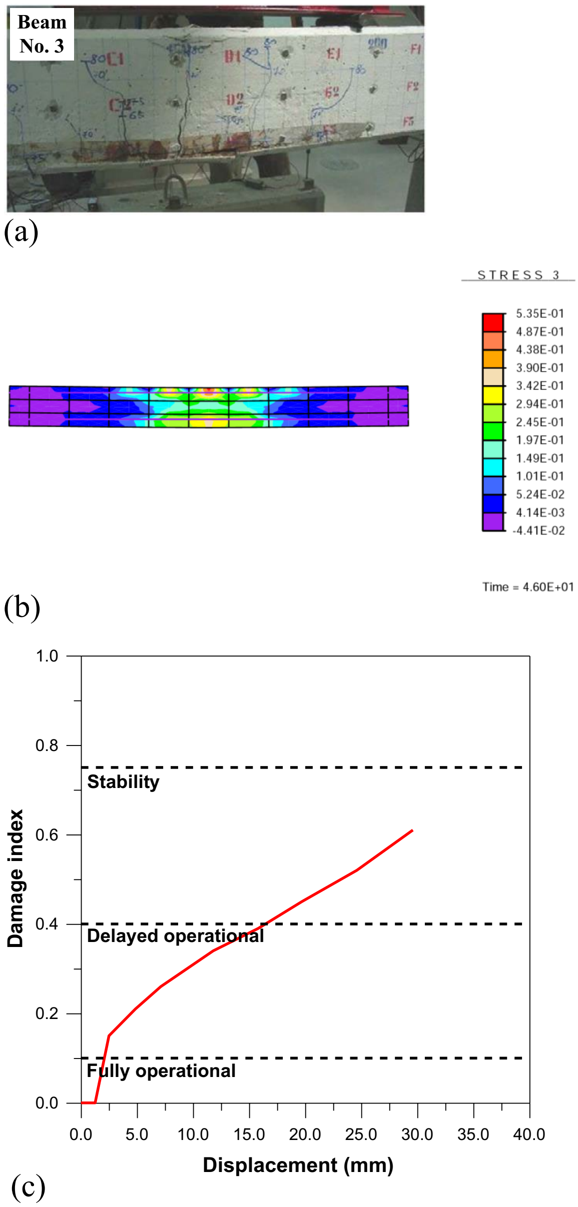


Fig. 13 Assessment of performance level for Beam No. 3: (**a**) experimental crack pattern at failure (Rinaldi et al., 2010); (**b**) analytical crack pattern at failure; and (**c**) analytical performance assessment using modified damage indices

in the field. Further developments are needed to incorporate the bond-slip of prestressing steel and reinforcing steel, including the stirrups corrosion and deterioration of bond strength.

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Author contributions
There is only one author in the current study. The author read and approved the final manuscript.

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Availability of data and materials
The research data used to support the finding of this study are described and included in the article. Furthermore, some of the data used in this study are also supported by providing references as described in the article.

Declarations

Competing interests
The author declares no competing interests.

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